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Recognition of the mechanical properties for soils in complex conditions: a case study

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Abstract

The aim of this paper is to discuss the shear resistance angle φ' of structurally complex formations in San Giuliano del Sannio (CB) for the construction of a strategic building. Large lithoid stones in a clayey-like matrix constitute the main soil formations. In this condition, it is doubtful how to evaluate φ' for geotechnical design, being very small the values obtained by conventional laboratory tests on the fine-graded matrix. Two alternative approaches could be suggested to detect the geotechnical parameters for design, specifically the φ' the angle: using the shear waves velocity and its empirical relationship with the soil resistance and through the back-analyses of an existing embedded retaining wall. It is shown that the two proposed pattern give more realistic values of the soil resistance for this kind of material, with respect to the ones obtained by conventional laboratory tests.

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Keywords: Soil complex conditions; shear resistance angle; in situ tests correlations; embedded retaining wall back-analysis

1. General information for the studying area

Flysch deposits are structurally complex formations composed of rocks and hard shales with disarranged or chaotic silty and marly clays. These materials, a common formation in Molise Region, have strong heterogeneity and anisotropy due to stratification, nodules, inclusions, joints and fractures, which affect the soil texture.

This paper will deal with the geotechnical characterization of a almost flat site (~1600 m^2) in the municipality of

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San Giuliano del Sannio (CB), about 10 km S of Campobasso. The site [1,2] presents a lithological sequence with highly unstructured and chaotic arrangement due to the presence of a thick layer of paleolandslide located on a contact strip between the calcareous-clayey and clayey-calcareous lithofacies of Varicolori, locally disarranged and/or mixed probably because of the overthrust. The site is located in a particular area characterized by tectonic interferences, where there are thicker layers of chaotic materials resulting from disruption and probable mixing of clay-limestone and limestone lithofacies of the Varicolori complex. Surveys data identify their overlap on mainly clayey-marly materials, in line with the hypothesis of the overthrust. Direct displacement systems and blankets of thick and irregular debris in earthy matrix, locally due to ancient landslides movements are individuated in the area.

This specific study focuses on the values of the shear resistance angle φ’ for soil. To this end, in the area there were executed a series of nine boreholes (up to a depth of 30 m from the ground level) together with SPT, Down-Hole tests, piezometers installation, sampling and laboratory tests. Common laboratory and in situ tests might do not produce results easy to apply for engineering design [3]. For instance, static penetrometric tests cannot be performed for practical problems (i.e., presence of stones or rock formation), while the results of dynamic penetrometric tests might be influenced by development of uncontrolled pore water pressures. Furthermore, for the lab. tests, the reliability of the results on small-sized specimens could be somehow questionable.

In the given area, a two stories building will be constructed using shallow foundation 1.4 m wide, placed at a depth of 1.2 m from ground level. Flexible retaining walls having an overall length of 9 m were built in December 2012 on two side of the site. The location of the boreholes, the wall and the building is shown in Figure 1, together with the two orthogonal cross sections of the area, underlining the main formations identified.

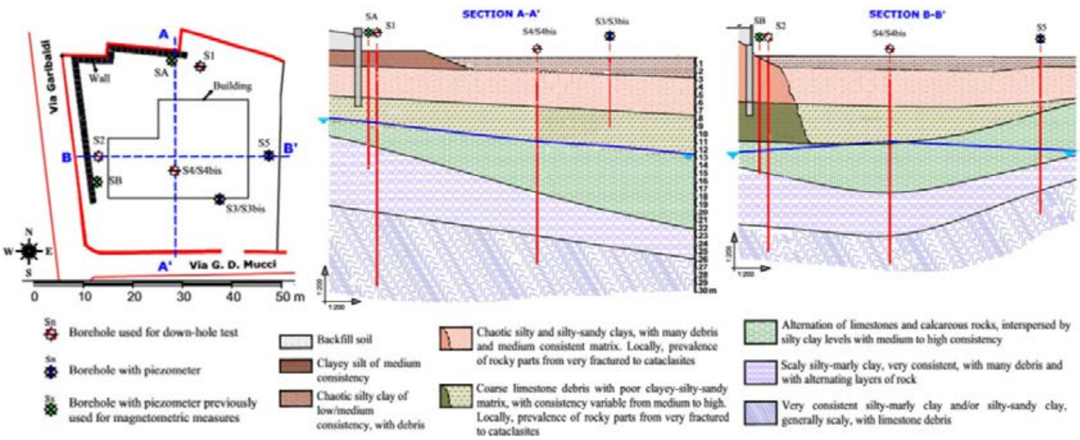

Fig. 1. Plan and simplified geological cross-sections of the test site in San Giuliano del Sannio (CB).

2. Laboratory Tests Results

Table 1 summarizes the shear resistance parameters as evaluated from set of conventional commercial laboratory tests on different samples, which included drained and undrained triaxial tests and annular ring shear tests. In the same table, M* indicates the slope of the straight line that interpolates the values of the peak strength in the plane <q-p'> by requiring the passage through the origin of axes. This value is converted in a shear strength angle φ* through the relationship $M^* = 6 \sin \varphi^* / (3 - \sin \varphi^*)$, imposing the absence of the effective cohesion. Figure 2a and Figure 2b show, instead, the available grading curves on the sampled soils and the Atterberg limits profiles. Two main types of materials can be seen: a group having a clear prevalence of coarse material that inhibits the execution of mechanical tests, the other constituted by fine grained materials with a relative homogeneity in the diameter of the particles. The plasticity is medium – high and the consistency is high on average, resulting the natural water content w less than or slightly greater than w_P. Overall, small values of the shear resistance angles emerge from the laboratory tests. This could be due to excessive disturbance during sampling or during specimens preparation, because of unknown mistakes in the experimental procedures, or because of the state of gravitational and/or tectonic rearrangement of materials.

Table 1. Summary table of the strength parameters obtained from laboratory tests.

				Triaxial test				Annular ring shear test
Borehole	Sample	Depth (m)	Note	c’ (kPa)	φ’ (°)	M* (-)	φ* (°)	φ’ (°)
S1	R2	7.80-8.40	Partially disturbed sample	15	14.5	0.65	17.2	10.5
S2	R1	22.0-22.6	Partially disturbed sample	35	13	0.67	17.5	9.3
S4	R1	10.2-10.65	Partially disturbed sample	10	13	0.61	16.0	11.3
S4	I2	14.2-14.5	Undisturbed sample	0	13	0.47	12.6	11.8

The values of the measured critical state shear resistance angle φ* are compared with literature data [4], as empirical function of the soils plasticity index (Fig. 2c). The resistance values are significantly lower than those measured for other soils. This aspect is even more relevant considering that literature values are for the critical state condition and those of San Giuliano del Sannio are, instead, indicative of the peak strength. The measured organic substance content on soil is always lower than 1%, a value that cannot justify alone the extent of the measured shear resistance values.

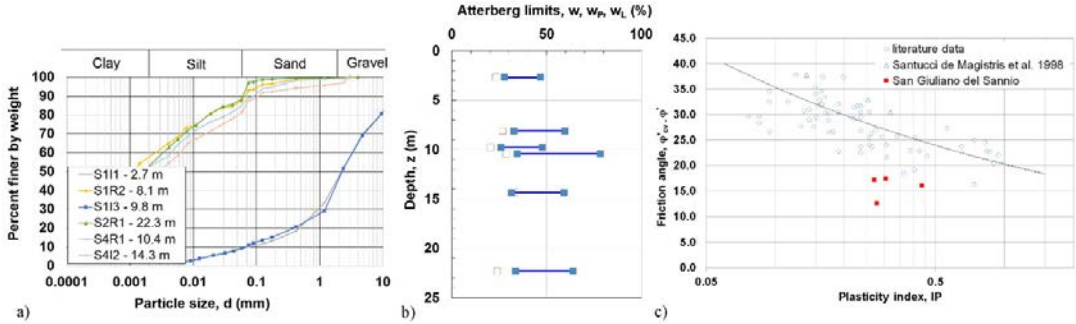

Fig. 2. a) Available grading curves and b) Atterberg limits and natural water content profiles for the test site; c) critical state resistance angle vs plasticity index: comparison between literature and this case study.

3. In-situ tests analysis

The available in situ tests (SPT and shear waves velocity measurements, Fig. 3a,b) could provide further insights about soils mechanical properties. A number of shots equal to 100 conventionally designates the refusal condition of the SPT test. It is possible to observe the high variability of the measurements and the difficulty in distinguishing clear stratigraphic passages. The simultaneous presence of finite values of the number of shots and refusal cases confirms the heterogeneous nature of the materials, which identifies lithoid portions interspersed with loose soil zones. The indications from the SPT tests allow an estimate, even approximate, of the shear strength of the penetrated formations. From the shear waves velocity profiles of the down-hole tests, it is noted the moderate stiffness of the materials, the local oscillation of the waves propagation speed starting from about 6m below the ground level, the differences between the data obtained from the S1 survey and those from S2 and S4 that are substantially overlapping. Referring to SPT tests, for relatively fine grain size strata (fine sand or silt) under groundwater level (found below 10m from ground level), when N_{SPT} was greater than 15, the number of shots has been corrected through Terzaghi’s formulation $N_{SPT,red} = (N_{SPT} - 15)/2 + 15$. This correction has been applied, as precautionary measure, in a limited number of cases. Being the measurement made using a cone rather than the open sampler, a conversion factor equal to one was assumed. The values of the shear resistance angles (Fig. 3c) have been calculated starting from N_{SPT} values using the correlations in [5], following the directions given in [6], and from some other correlation reported in [7] (Table 2). The adopted formulation required the $(N_1)_{60}$ corrected N_{SPT} value and, for this purpose, the approach given by [8] was employed $(N_1)_{60} = \sqrt{p_a/\sigma'_v}\,N_{60}$, being N_{60} equal to N_{SPT} in our case. Such correlations were obtained in a geolitologhical context different from that of the Molise Region. However, as far as the Authors know, no other approaches are currently available to obtain the shear resistance from SPT blowcount.

Table 2. Some correlations between the SPT blow number and the angle of shear resistance (modified after [9]).

CODE	REFERENCE	EQUATION
SCH	Schmertmann (1975) in Kulhawy and Mayne [6]	$\varphi' = \tan^{-1}\left[N_{SPT}/\left(12.2 + 20.3\frac{\sigma'_v}{p_a}\right)\right]^{0.34}$
PHT - KM	Peck et al. (1974) in Kulhawy and Mayne [6]	$\varphi' \approx 54 - 27.6034\exp(-0.014\,(N_1)_{60})$
HU	Hatanaka, Uchida (1996)	$\varphi' = \sqrt{20(N_1)_{60}} + 20$ for $3.5 \leq (N_1)_{60} \leq 30$
PHT - W	Peck, Hanson, Thornburn (1974) in Wolff (1989)	$\varphi' = 27.1 + 0.3(N_1)_{60} - 0.00054{(N_1)_{60}}^2$
MAY	Mayne et al. (2001)	$\varphi' = \sqrt{15.4(N_1)_{60}} + 20$
JRA	Japan Road Association (1996)	$\varphi' = \sqrt{15(N_1)_{60}} + 15$ for $(N_1)_{60} > 5$ and $\varphi' \leq 45°$

Concerning down-hole tests, the relationship between the shear waves velocity, Vs, and the number of blows of the SPT test has recently been studied [9], with reference to structurally complex formations of the Molise region. The proposed correlations are distinct on the basis of the main macro-lithologies present in the region: structured deposits, loose and altered soils. For the purpose of this study, the more conservative relationship - that for structured deposits - has been inverted, to estimate N_{SPT} values starting from the shear waves velocity profiles obtained through DH tests: $N_{SPT} = (V_s/104.55)^{1/0.385}$. These data, although obtained indirectly, allow further approximate estimates of the shear resistance angle of the materials, using the same formulations previously mentioned. Results of the computations are in Figure 3d. In converting V_S in N_{SPT} further incertitude arises. However, the correlation is calibrated using local soils and the measurements of shear wave velocities are not much affected by local heterogeneities as N_{SPT} does.

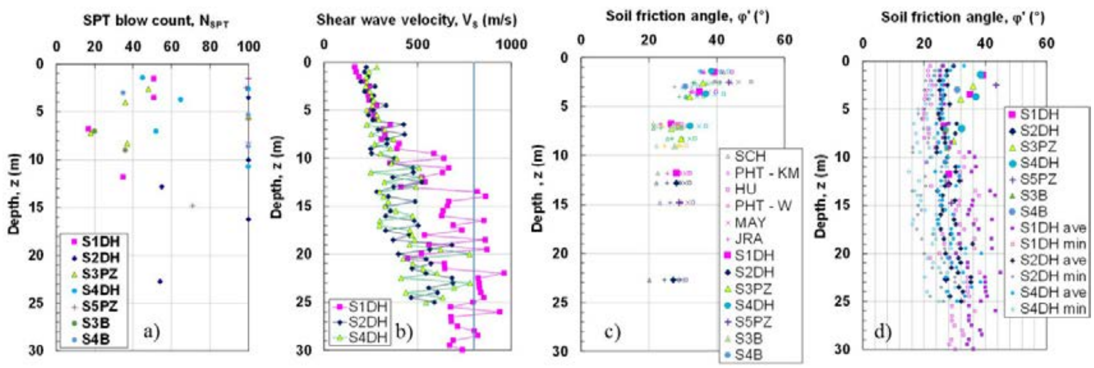

Fig. 3. Data from SPT (a) and DH tests (b) and soil friction angle profile from SPT (c) and after Vs into NSPT conversion (d). In this case, larger symbols are the average values, while smaller symbols are minimum and mean values derived from Vs.

From SPT results, it can be observed a lack of homogeneity of the resistance values with depth, which assume highest values for the surface formations. This last statement may partly be related to the nature and manner of execution of the standard penetration test. Thus, even if the lithoid component of the formations in the area is not very sensitive to the geostatic confinement stress state, it may be passed through more easily from the test tool if localized in the surface part of the deposit rather than in the deepest part. In this hypothesis, the resistance of the more superficial materials would take into account also, partially, of the rocky component presents in the site, while that of the deeper materials would be representative of the loose component. In any case, despite the more conservative formulations used to convert the number of blows of the SPT tests in soil resistance, this last one is clearly higher than laboratory tests values. From down-hole tests, it is possible to observe how the shear resistance angle values, derived by indirect means, are generally lower than those obtained through the measurement of the number of blows to advance the standard penetrometer. However, even the minimum values obtained in this way appear to be greater than the informations reachable through the interpretation of laboratory triaxial tests performed on samples. It is important to emphasize, once again, that resistance measurements obtained from DH tests are only approximate estimations, as well as to underline the capacity of the geophysical tests in providing a mechanical indicator that allows estimating the behavior of soils when characterized by clay horizons mixed with coarse limestone debris. All the in situ tests correlations confirm the doubts on the meaning and usefulness of the mechanical laboratory investigations results for the geotechnical characterization of the soil deposit in the area.

4. Retaining wall back-analysis

For the purposes of validating the geotechnical characterization of the most superficial soils, a preliminary parametric and simplified analysis of the retaining wall, looking at the equilibrium in the static and pseudo-static conditions, has been done. The structure seems to be verified when higher values of soil strength parameters, compared to the findings from laboratory investigations, are used for the analysis. This confirms the need of a global rather than punctual measurement for the mechanical characterization of the soil deposit. It is assumed that the structure is above the groundwater level, the total height (H) is 9m, the height above the ground (h) is 3m, d is the embedding depth of the wall and the soil is homogeneous referring to the mechanical characteristics of the analysis.

The static analysis was conducted using the full method and the Blum's method [10], adopting the theories proposed by [11] for evaluating active K_A and passive K_P pressure coefficients. A couple of combinations for soil-wall friction values were adopted: those of the current practice ($\delta_A = 2/3\ \varphi'$; $\delta_P = 0$) and those ($\delta_A = 2/3\ \varphi'$; $\delta_P = 1/2\ \varphi'$) suggested by [12]. In Figure 4 is plotted the relation between the d/h ratio and the shear resistance angle obtained for the two different methods, adding the values derived from the empirical formulation proposed by [13] with a unit safety factor. Being the ratio d/h equal to 2, it is clear that the wall is in equilibrium when the soil interacting with it can be modeled as a granular medium with shear resistance angle between 15.5÷18.5°. The equilibrium conditions of [13] are not considered in this comparison because they are derived from experimental cases of d/h ratio smaller values. In conclusion, the back-analysis leads to φ' mean values for the upper 9m of the subsoil at least equal to ~16°, according to the simplified assumption mentioned before. Smaller values would bring the wall to collapse.

Considering that the wall appears stable at the moment of writing this note, it might be useful to extend this back-analysis adding the seismic forces that could have hit the structure since it was build (December 2012) to now. To this end, all the recent earthquakes with a moment magnitude $M_w \geq 3$ occurred in a radius of 30 km from San Giuliano del Sannio were collected through INGV website, encompassing 17 earthquakes including a moderate $M_w = 5$ shake. This event, according to INGV ShakeMap, produced a peak acceleration at site in the order of 0.012 g, because located at an epicentral distance of about 19 km. Using combination of magnitude and epicentral distance, through the simple ground motion prediction equation in [16], the January 16, 2016 $M_w = 4.3$ earthquake should be the one producing the largest peak ground acceleration at site. However, such acceleration is quite low, being in the order of 0.0095 g from INGV Shakemap. Both earthquakes are thus not able to significantly modify the stability condition of the embedded wall. However, the stability of the wall should be ensured even under the design earthquake. Following [17], for this class of construction it should be assumed a nominal life of 100 years and a usage coefficient 2. Considering, for instance the ultimate limit state for the life safeguard SLV, the design earthquake should have a returning period of T_R 1898 years. In this case, the forecast peak acceleration for horizontal outcropping bedrock is $a_g = 0.434g$.

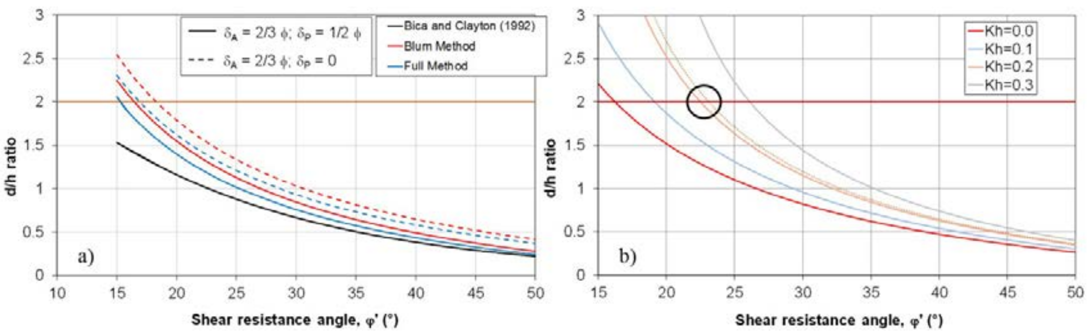

Fig. 4. Normalized limit embedding depth vs shear resistance angle for homogeneous soil: a) static case (modified from [14], b) pseudo-static case (modified from [15]).

A simple pseudo-static analysis of the wall can be executed using the Blum's method and a horizontal seismic coefficient $k_h = \alpha\,\beta\,Sa_g/g$. Seismic active and passive pressure coefficients, respectively K_{AE} and K_{PE}, can be computed through Mononobe-Okabe theory and formulations in [18], assuming $\delta_A = 2/3\ \varphi'$ and $\delta_P = 1/2\ \varphi'$. The coefficient α is 1 because of the limited height of the wall, β is 0.5 for a top structure maximum displacement ≤ 0.05m, and the amplification factor S is 1 for the topography T1 and site class B. In the hypothesis of rigid-plastic behaviour of the soil, no overload, lack of groundwater, homogenous subsoil and Blum's approximation, it is possible to define the relationship between d/h ratio and shear friction angle wall (Figure 4b) for different seismic horizontal coefficients k_h. For the d/h ratio equal to 2, the wall is stable against seismic loading only if the soil has a minimum design shear resistance angle value of 23°.

5. Conclusion

The geological complexity of San Giuliano del Sannio (Molise) is documented in existing geological studies related to the seismic microzonation of the area and to the IFFI landslides project, but is also confirmed by the in situ and laboratory tests results, characterized by high complexity. The insights led to achieve a good level of knowledge of the geological and stratigraphic conditions, but leave still doubts concerning the actual geotechnical behavior of soils, particularly following the outcome of the apparently abnormal laboratory tests. This deficiency strongly inhibits the definition of a geotechnical model sufficiently compatible with geological problems detected in the area. To clarify the significance of the uncertainties for the shear resistance angle values, comparison between laboratory tests, in situ tests and retaining wall back-analysis have been done. It appears that the mechanical characteristics of the materials in the area are better than those found by laboratory tests, probably because the geological complexity does not allow the formations global characterization through the single mechanical response of the finest components, and because of some possible error in the experimental procedures employed in laboratory. Moreover, the modest strength which emerges from triaxial tests could be due to the strong rearrangement that soils have suffered during their geological history and their particular mineralogical and structural nature.

References

[1] IFFI, Inventario dei Fenomeni Franosi in Italia, ISPRA - Dipartimento Difesa del Suolo, Servizio Geologico d'Italia – Regione Molise, 2006.

[2] Regione Molise, Documenti di accompagnamento agli studi di microzonazione sismica del comune di San Giuliano del Sannio, 2007.

[3] L. Picarelli, C. Di Maio, L. Olivares, G. Urciuoli, Properties and behaviour of tectonized clay shales in Italy, 2nd Int. Symp. on The Geotechnics of Hard Soils - Soft Rocks, Napoli, 1998.

[4] F. Santucci de Magistris, F. Silvestri, F. Vinale, Physical and mechanical properties of a compacted silty sand with low bentonite fraction, Canadian Geotechnical Journal 35(6) (1998) 909-925.

[5] J.H. Schmertmann, Measurement of In situ Shear Strength, Proceedings of Conference on in situ measurement of soil properties, North Carolina State University, ASCE 2 (1975) 57-138.

[6] F.H. Kulhawy, P.W. Mayne, Manual on Estimating Soil Properties for Foundation Design, EL-6800, Electric Power Research Institute, 1990.

[7] NCHRP, LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures, Transport. Res. Board, Report 651, 2010.

[8] S.S.C. Liao, R.V. Whitman, Overburden Correction Factors for SPT in Sand, Journal of Geotechnical Engineering 112(3) (1986) 373–377.

[9] S. Fabbrocino, G. Lanzano, G. Forte, F. Santucci de Magistris, G. Fabbrocino, SPT blow count vs. shear wave velocity relationship in the structurally complex formations of the Molise region (Italy), Engineering Geology 187 (2015) 84–97.

[10] British Standard, Code of practice for Earth retaining structures, BS 8002:1994, 2001.

[11] R. Lancellotta, Analytical solution of passive earth pressure, Geotechnique 52(8) (2002) 617-619.

[12] C.J. Padfield, R.J. Mair, Design of retaining walls embedded in stiff clays, Construction Industry Research and Information Association, Report No. 104, London, 1984.

[13] A.V.D. Bica, C.R.I. Clayton, The preliminary design of free embedded cantilever walls in granular soil, Proc. Int. Conf. on Retaining Structures, Cambridge, 1992.

[14] C. Visone, F. Santucci de Magistris, A review of design methods for retaining structures under seismic loadings, Strategies for reduction of the seismic risk, in: G. Fabbrocino and F. Santucci de Magistris (Eds.) (2008) 51-64.

[15] C. Visone, F. Santucci de Magistris, Performance-Based approach in seismic design of embedded retaining walls, Proc. XVII ICSMGE, Alexandria, 2009.

[16] F. Sabetta, A. Pugliese, Estimation of Response Spectra and Simulation of Nonstationary Earthquake Ground Motions, BSSA 86(2) (1996) 337-352.

[17] CS.LL.PP., Norme tecniche per le costruzioni, G.U.R.I. 29, DM 14 Gennaio 2008.

[18] R. Lancellotta, Lower-bound approach for seismic passive earth resistance, Geotechnique 57(3) (2007) 319-321.